9.1.1 GENERAL

Live load deflections shall be less than 1/800 of the span, or less than 1/1000 of the span for bridges with pedestrians. (C1)

Refer to Subsection 8.2 for deck overhang limitations.

Fully bonded internal prestressing shall be used for at least the portion of the prestressing steel needed for ultimate strength. (C2)

Partial prestressing may be used for repairs or upgrades to existing structures. The criteria for strength and allowable compression stress in Section 9 of the AASHTO Standard Specifications shall apply to partially prestressed members. The allowable tension stresses in Section 9 may be waived in lieu of the crack control provisions in Section 8 (under Serviceability Requirements, Distribution of Flexural Reinforcement). (C3)

If strand is used, the plans shall be based on the use of low-relaxation strand. (C4)

The design of curved T-beams with any horizontal curvature, and curved box girders with a radius less then 240 m (800 ft.), shall consider curvature effects such as torsion, lateral flange bending, duct blowout, lateral web bending, shear redistribution from skew (since skew can combine adversely with curvature effects), and increased load distribution to the outside webs. Any diaphragm requirements due to curvature shall also be considered. (C5)

C1: AASHTO does not give deflection limits in the prestressed concrete chapter. The values given here are taken from the AASHTO chapters on structural steel and reinforced concrete.

C2: Fully bonded internal prestressing should be used whenever possible. Doing so improves overload behavior at operating levels for both flexure and shear, and improves crack control. However external post-tensioning and unbonded strands are occasionally needed. External post-tensioning is used for repairs and segmental girders. Unbonded strands are used for temporary tensioning, to provide for future post-tensioning, and to control stresses in pretensioned members by means of sleeving. This policy provides a limit for such practices.

C3: Adding prestressing to existing structures can improve serviceability and reduce cracking. This policy allows such tensioning which may otherwise be prohibited due to a lack of code provisions for partial prestressing.

C4: There has been insufficient use of stress-relieved strand to justify continuing our previous policy of allowing the Contractor the option of either stress-relieved or low-lax strand. Low-lax strand will normally be slightly more efficient to use and have more predictable deflections.

C5: The CDOT Staff Bridge Worksheets for box girders (618-1

Curved webs with any horizontal curvature shall have cross ties at each level of ducts, a minimum of #10M at 350 mm (#3 at 14”). (C6)

Maximum stirrup spacing shall be 450 mm (18”). Minimum shear steel shall be at least Av=0.93(b’)/fy (square mm per mm), where b’ is the web width in mm and fy the reinforcement yield strength in MPa {=135(b’)/fy (square inches per inch) where b’ is web width in inches and fy is in psi}. Webs for a distance d in front of anchorages and integral caps shall have at least double this minimum reinforcement. (C7)

The minimum side face steel located in webs shall be 1.5 times the above minimum shear steel area specified for areas more than distance d from supports, and spaced at 300 mm (12”) maximum. This steel shall be used throughout the length of cast-in-place members (including cast-in-place segmental), and shall be located in at least the end portion of precast members. (C8)

through 618-3) were checked for curvature problems at radii greater than or equal to 240 mm (800 ft.) with a jacking force no greater than 5280 kN (1187 kips) per duct. Curved T-girders may present web lateral bending problems at ultimate strength.

C6: These cross ties help arrest “unzipping” if duct lateral blowout is initiated by a construction flaw. Normally tendons should not be curved so sharply that the concrete alone cannot resist the blowout forces at ultimate tendon strength using the ultimate concrete tensile strength or punching shear strength.

C7: This minimum stirrup reinforcing matches our historical practice in Colorado. It provides stirrups that are adequate as temperature and shrinkage steel. It helps control the size of shear cracks, because this amount of reinforcing ensures that the member’s cracked shear strength is greater than the shear necessary to crack the section. This minimum also overcomes uncertainty about adequacy of AASHTO’s 0.345 MPa (50 psi) requirement with high strength concrete. The occasional need to control bursting forces which extend ahead of the typical anchorage block or abutment indicates a need for more stirrups ahead of anchorages. The lack of support induced vertical compression may induce a similar need at integral caps. We have had a few bridges with poorly controlled horizontal cracks in webs ahead of anchorages to indicate this problem.

C8: This provides distributed horizontal steel to help control cracks, which may include the nearly vertical shear cracks which occur near member ends, temperature or shrinkage cracks, cracks due to formwork or shoring settling, or flexural cracks at overload.

Note, Colorado allows trucks that weigh up to 91 Metric Tons (200 kips) to use bridges on a routine basis, if the bridges have an adequate operating capacity. Though usually much less for actual load

The negative moment zones of continuous bridges shall be designed for shear by the latest AASHTO method and not by the method given by the 1979 Interim of the AASHTO Standard Specifications. (C9)

The contract plans for post-tensioned members shall specify:
- jacking force
- area of prestressing steel
- minimum concrete strength at jacking and at 28 days
- center of gravity of prestressing force path
- jacking ends
- anchor sets
- friction constants
- long term losses assumed in the design
- strand and duct size assumed in the design
- net long term deflections and expected cambers (C10)

The contract plan for pretensioned members shall specify:
- jacking force
- area of prestressing steel
- minimum concrete strength at jacking and at 28 days
- center of gravity of prestressing force path
- final force at the critical section
- net long term deflections and expected cambers (C10)

The design shall be based on a maximum jacking force of 75% of the ultimate strength of prestressing strands. (C11)

All mild steel shall have at least 50 mm (2”) clear between parallel bars, including spirals. (C12)

distributions, the calculated load may be up to 77% of the bridge members’ ultimate strength. Normal code provisions may not adequately control flexural and shear cracking for routine excursions to these stress levels over the design life of the bridge.

C9: The 1979 interim methodology assumes simple span behavior. This may be unconservative for our precast girder bridges made continuous by integral pier diaphragms. Continuity can result in greater shears at continuous supports than is predicted by simple span analysis; and, large bending moments. The flexure cracks from these bending moments may propagate into the web under overloads, reducing the shear capacity.

Most often, however, the 1979 Interim methodology results in much more shear reinforcing than is required by the current AASHTO specifications.

C10: This policy provides a standard and consistent method for detailing prestressing. It also provides maximum flexibility to contractors and fabricators. In unusually difficult situations, the data for each tendon may need to be specified.

C11: This limit provides a margin for the correction of field problems, increased safety, and reduced strand breakage.

C12: This provides access for a vibrator. The segmental bridges at Vail Pass had problems with concrete consolidation at tendon anchorage’s when this requirement was not met. Suppliers often specify spirals with a pitch, which will not meet this requirement. Consequently, shop drawings need to be checked for this clearance.

Immediately after tensioning, extreme fiber tension shall be less than 1.4 MPa (200 psi); except, portions of the extreme fiber that are not subject to tension under full service load (after all losses have occurred), or are not intended to be prestressed, may have tension up to $0.62\sqrt{f'ci}$ MPa ($7.5\sqrt{f'ci}$ psi) if well distributed steel is present to carry the tension. (C13)

Under full dead load, without live load and after all losses, no part of the top or bottom fiber which resists moments using prestressing shall be in tension. (C14)

Under full loads, after losses, tension due to live load will be permitted in the extreme fibers of prestressed parts of members if well bonded well distributed steel (prestressing included) is provided to carry the tension. (C15)

If any part of the top of a deck resists moments using prestressing, the tension in that part shall not exceed $0.25\sqrt{f'ci}$ MPa ($3\sqrt{f'c}$ psi). (C16)

9.1.2 CAST-IN-PLACE OR POST-TENSIONED

The f'c shall be at least 30 MPa (4500 psi) when any part of the prestressed member forms any part of the deck. For cast-in-place members the required f'c shall not be greater than 40 MPa (5800 psi). The required f'c shown in the plans shall be equal to or greater than the f'ci required. Either concrete Class D (30 MPa {4500 psi}), Class S35 (35 MPa {5000 psi}) or Class S40 (40 MPa {5800 psi}) shall be used, listed here in order of preference. (C17)

C13: These limits are from the AASHTO Standard Specifications. They help prevent cracking and distress from tensioning stresses.

C14: This ensures that live load cracks caused by overloads will close.

C15: In contrast to no tension being allowed under final dead load, live load tension is allowed to economize designs. It is not our intent to apply compression or tension limits to mild reinforced decks that are not pretensioned or post-tensioned.

C16: This provides for less deck cracking and presumably less deterioration from salt intrusion. This provision is intended for portions of decks that are pretensioned or post-tensioned.

C17: For cast-in-place concrete 6000 psi maximum has been the Department's standard practice. This has been hard converted to 5800 psi for the Department's migration to using metric units. There is typically less variation in the quality of concrete at lower strength, and lower strength concrete can be more economical, consequently Class D should be assumed initially for design. If greater strength is needed, then Class S35 should be tried, or as last preference, Class S40. In 1997 Staff Bridge with Staff Materials decided to regulate cast-in-place superstructure concrete to only three different strengths to help reduce the variations in mix designs that the Department was receiving. If the need arises, we may develop higher strength classes in the future. If higher strength in needed for a project, the Staff Bridge Engineer shall be consulted.

The plans shall show the configuration (arrangement) of the anchorage's, and the arrangement of ducts at typical high and low points which are appropriate for the duct and strand size noted on the plans. The arrangement of anchorages shall permit a center to center anchorage spacing of at least $\sqrt{(2.2(P_j)/(f'_{ci}))}$ meters (inches), and a spacing from the center of each anchorage to the nearest concrete edge of at least half that value. If web flares are needed for this arrangement, they shall be dimensioned in the plans and included in the quantities. (C18)

The post-tensioning arrangement provided by the plans shall permit the use of either 13 mm (0.5") or 15 mm (0.6") strands. (C19)

The design shall not require the use of more than 5280 kN (1187 kips) of jacking force per duct. (C20)

Ducts shall be spaced at least 44% of the duct diameter or 38 mm (1.5") minimum clear from each other, whichever is greater. (C21)

Cast-in-place webs shall have a clear space between ducts and formwork, and between longitudinal rebar and formwork, of at least 75% of the nominal duct diameter, but not less than 75 mm (3") to facilitate concrete placement and vibrator use. At least 50 mm (2") clear should be provided between post-tensioning ducts and the outside face of precast girder webs. (C22)

C18: This requires the designer to provide a practical solution to arranging the post-tensioning in the contract plans. The designer's solution should not require a strand steel area greater than 40% of the duct inside cross section area for bundles of strands. 33% to 37% duct fill is typical. More area may be required for long ducts of the smaller diameters (under 3.5"). The combination of maximum jacking force per duct (at 75% of ultimate) and duct size should be one provided for in the current literature of one of our common suppliers of post-tensioning components, such as DSI or VSL. Alternative arrangements may be proposed by the supplier on the shop drawings.

C19: This improves competition.

C20: This maximum improves competition and it is consistent with established practice. Previously this limit was 3716 kN (835 kips). The current limit of 5280 kN (1187 kips) is reflected in CDOT Staff bridge Worksheets 618-1 through 618-6. The designer can approve shop plans with a somewhat higher jacking force per duct if it does not cause any problems. Note, the 30" thickness of CDOT's typical integral abutment is marginal for containing the bursting forces and spirals needed for this maximum jacking force.

C21: This facilitates concrete placement and helps prevent problems in curved area. The increase for 4" and larger ducts is due to recent consolidation problems with larger duct diameters.

C22: This facilitates concrete placement, vibrator access, and reduces weakened plane cracking in thin webs.

Cast-in-place concrete superstructures shall be considered during the structure selection report process. T-girders, spread box girders, full width box girders, and slabs should be investigated. The investigation should be made with structure depth, web size and web spacing optimized for each type of superstructure. (C23)

9.1.3 PRECAST OR PRETENSIONED

The f'ci for precast girders shall generally be limited to 45 MPa (6500 psi) and f'c to 60 MPa (8500 psi). These limits may be increased by up to 7 MPa (1000 psi) if the feasibility of efficient production (i.e., no net increase in costs) for the particular project with these strengths has been confirmed with our usual fabricators. The f'c shall not be less than 30 MPa (4500 psi). The required f'c shown in the plans, shall be equal to or greater than the f'ci required. (C24)

C23: T-girders may be less expensive than boxes in situations where the strength contribution of the bottom slab does not out weigh its cost and dead load.

The minimum web width is 250 mm (10") for 100 mm (4") diameter ducts and 290 mm (11.25") for 115 mm (4.5") ducts. These minimum widths should be used for short span situations that do not require large girder depths, large quantities of tensioning (i.e., more than two ducts per web), nor close web spacing. Otherwise 380 mm (15") wide webs should be used to allow placement of two ducts per row (staggered) and easy concrete placement.

Very long span cast-in-place box section may be an exception when the desired prestress eccentricity at mid-span for final loads cannot be used due to mid-span negative moments that occur during the deck pour. In this case 250 mm (10") webs may help control dead weight; however, 380 mm (15") may still be effective at piers where the shears are high and larger prestress eccentricity can be used.

Webs should normally be placed as far apart as practical to minimize web concrete and, especially, formwork costs, though deck costs must also be considered. 3600 mm (12') clear spacing between webs should not be considered exceptional.

C24: Previously the f'c limit for precast concrete was 54 MPa (7500 psi). This was changed to 60 MPa (8500 psi) to meet the current capability of our precast suppliers. Recent changes (1996) to the AASHTO code allowing higher service compressive stresses will typically make increasing the f'c limit unnecessary.

The higher liveload (HL93) in LRFD may make values of f'c greater than 8500 psi useful for some LRFD designs, especially with the new U girder sections which have a variable width top flange that can

Using lump sum losses for precast pre-tensioned girders is discouraged. If lump sum losses are used for precast pre-tensioned members, the tension in the extreme fiber shall be limited to $0.25\sqrt{f'c}$ MPa ($3\sqrt{f'c}$ psi). (C25)

End blocks shall be used for box girders. End blocks are not required for typical applications of the Colorado BT-girders using the CDOT Staff Bridge Worksheet details. (C26)

Composite precast pre-tensioned girders spliced at pier diaphragms shall typically be designed as simple spans, with reinforcing provided for the positive and negative moments resulting from continuity at and near the piers. Alternatively, this and other arrangements of spliced girders may be designed taking continuity into account if the necessary additional design considerations are conducted. When utilizing continuity for the girder design, the effects of differential shrinkage, differential temperature, and any redistribution of moments due to creep shall be investigated.

The design of precast girders should not be made dependent on continuity, or require post-tensioning, unless doing so significantly reduces the cost of the structure. For simple spans made continuous the beneficial effects of continuity on girder design should not be used unless the number of girder lines is reduced. (C27)

be adjusted to reduce weight if higher strength concretes are used.

The f'ci affects economy by dictating how long girders must remain in the casting beds. Due to improvements in fabrication practices and technology, using f'ci of 50 MPa (7000 psi) is probably practical now for limited production, and 45 MPa (6500 psi) for routine production, except in very cold weather or for large numbers of precast box girders which may need a more fluid mix than those mixes which provide the highest early strengths.

C25: We seldom use lump sum losses for precast members. Detailed losses for the sections we normally deal with indicate that the use of lump sum losses can be unconservative. The reduced allowable stress given here helps correct this.

C26: Without end blocks, the previously used Colorado G-girder sections may have had inadequate shear, bursting, and handling strengths. Our BT-girder sections have thicker webs and bar details for the associated problems and therefore do not require end blocks for ordinary usage. Adding post-tensioning anchorage's to the BT-girder is an instance where end blocks may be useful.

C27: Previously all precast pre-tensioned girders were required to be designed as simple spans. This was due to the accuracy of commonly used methods for determining the concrete stresses resulting from the primary and secondary effects of shrinkage, temperature, temperature differentials, and creep. Due to the information now available to address these issues, and to take advantage of the economy offered by continuity, in 1993 the Department began to allow continuity to be used as approved on a case-by-case basis. In 2002, this policy is further liberalized, however it is important that designers not make their design dependent on continuity, or on post-

Post-tensioning may be used with precast girders provided the staged and long term effects of the tensioning are adequately accounted for in the design. Post-tensioning may be used to optimize the design of long span girders, to facilitate splicing girders, or to optimize the fabrication process. Fabricators may be allowed the option of providing a part of intended pre-tensioning with post-tensioning. (C28)

Girder haunches shall be sized so no design changes or deck rebar shifts will be needed if the predicted camber plus the girder depth given in the plans is exceeded by 38 mm (1.5”) before the deck pour. (C29)

tensioning, unless there is a significant benefit in costs.

Note, In April 2002 CDOT’s policy was changed to use continuity for the rating of all precast girders, regardless of the method of design, to provide uniformity in the inventory for the operating rating for moment of these girders.

C28: Post-tensioning has been used in combination with pre-tensioning for splicing long span BT-girders and for providing the necessary tensioning when the jacking force exceeds fabricator bed capacity. The latter may be necessary to use the new BT sections efficiently, as up to 10000 kN (2250 K) of jacking force is needed to fully utilize the concrete capacity provided by these sections. However, our usual fabricators have taken steps to improve their jacking capacities to well beyond 10,000 kN (2250 K). Allowing post-tensioning to be substituted for intended pre-tensioning should be avoided if the fabricator has the ability to provide the necessary jacking forces with pre-tensioned steel only. Economics favor using pre-tensioning instead of post-tensioning, and as few post-tensioning stages as practical.

C29: The 38 mm (1.5”) required here has typically been enough tolerance to cover the unreliability of camber predictions and girder depth variations. However, as we extend our span length capability, or use shallower sections or new suppliers, more tolerance or better predictions may be needed. For additional information on camber and fabrication tolerances see PCI MNL-116. PCI MNL-116 allows +25 mm (+1”) camber tolerance for typical depth/span ratios, and +13 mm (+0.5”) girder depth tolerance.

Most of our inadequate haunch depth problems have been due to long delays between girder fabrication

It is the designer's responsibility to verify the constants used for camber prediction by the girder design software. A sensitivity analysis is recommended, and adjustment of the constants is required, as necessary to ensure camber predictions are within the 1.5" tolerance provided in the haunch calculations. (C30)

The average minimum haunch depth due to cross-slope plus the minimum haunch due to precast deck panels (25 mm {1"}) may be used for section properties. A weighted average haunch depth may be used for dead load calculations. The weighted average haunch shall be based on a girder camber no larger than the value shown in the plans. All other dimensions (haunch depth at the ends of girders, dead load deflection, and deck geometry) shall be from values shown in the plans. (C31)

and deck placement, and inadequate allowance for deck geometry. The 38 mm (1.5") should not be relied on to solve these problems. Long delays are addressed by a note in the plans alerting contractors to monitor camber growth, and deck geometry must be addressed during design as part of the girder haunch depth calculations.

Note that camber is sensitive to the prestressing path and may be controlled to a degree by adjustments to the path during design.

C30: CDOT's recent use of the conspan software for girder design has led to camber predictions that have not been tailored for local experience or practices. More recently, the use of Opis/Virtis software has been initiated. Designers need to become familiar with the methodology used by these applications for camber prediction and make the necessary adjustments to ensure the haunch depth and deflections used for design, and shown in the plans, is adequate.

C31: Previous practice had been to treat haunches conservatively by not using them for section properties and overestimating their dead load effect. This can be overly conservative when using BT-girders and precast deck panels, both of which result in significantly larger haunches than used in the past.

(d1+10(d2)+d3)/12 is a calculation for the weighted average haunch for dead load where the haunch depth at centerline of girder is d1 over one bearing, d2 at mid-span, and d3 over the other bearing. In most situations this provides a suitably accurate result for mid-span moment. This equation is derived for the mid-span moment affect assuming the haunch varies parabolically with the apex (either concave or convex) at mid-span.

The transverse reinforcing steel area in precast box girder flanges shall, as a minimum, be equal to the minimum required shear reinforcing steel for one web. (C32)

Precast girder segment joints shall be bonded with epoxy or with concrete closure pours. (C33)

The G-series girders have been discontinued and should not be used except for replacement of damaged G1730 (G68) girders and resetting existing girders. The G1370 (G54), G1730 (G68) and G1830 (G72) have been replaced as the Department's standard sections by Bulb-T girders: BT1070 (BT42), BT1370 (BT54), BT1600 (BT63), BT1830 (BT72) and BT2130 (BT84).

It is the designer's responsibility to verify stability of the girders during construction, especially the stability of exterior girders. Additional diaphragms, or modifications to CDOT's standard diaphragm details (see worksheet B-618-DF) may be needed for special situations; e.g., unusually large overhangs. Additional diaphragms, or modifications to the standard details, should not be used unless determined necessary by calculation. (C34)

CDOT now requires the LRFD specifications to be used for new structures. Until confidence in the LRFD specifications and software applications is achieved, designers will compare the results of LRFD with LFD. Differences should be expected. It is not a requirement that structures meet the requirements of LFRD as well as LFD specifications. It should be expected that various aspects of LRFD will be more, and others less, conservative than LFD. When LFD seems to indicate that an LRFD design is not adequate, the designer should verify the cause of the difference to assure no mistakes have been made.(C35)

C32: This policy helps ensure that the torsional shear strength and strand confinement, which may be needed, is provided.

C33: This practice improves waterproofing, and improves overload behavior in both flexure and shear.

C34: The BT series of girders are heavier and provide wider flanges, improving their stability during the construction stages. The notes in the worksheet (B-618-DF) provide for those situations where diaphragms are needed for additional stability against wind loads during the construction stages if full flange width leveling pads are used. The girders have been checked for stability during the deck pour when they have typical reasonable overhangs.

Designers should check that the resultant of construction loads falls within the area of the leveling pad and that the compression in the pad is less than the allowable strength (typically >2250 psi ultimate). Reasonable safety factors should be used for this check; e.g., by using the AASHTO LRFD load factors when using the ultimate strength of the leveling pad.

If the resultant falls outside of the pad, or the compression strength of the pad is exceeded, additional diaphragms should be provided to reduce eccentricity by causing the girders to overturn in concert. Improved moment connections between the diaphragm and girder (by modifying the standard connection details or using deeper diaphragms or bracing) may also be used to provide moment resistance and thereby reduce the eccentricity on the pad directly.

C35: The following factors may be expected to cause some aspects of LRFD designs to seem less conservative than prior LFD designs:

The calibration for LRFD Service III tensile stresses is likely to have

LRFD exterior girder distribution factors are invariably larger for exterior girders than interior, reversing the typical situation under LFD. To balance exterior girder designs with interior girder designs, overhangs should generally be limited to less than half the interior girder spacing. (C36)

less effect than CDOT's recent conservative use of HS25 for this serviceability check. See the LRFD commentary on service III.

LRFD sometimes uses more rational distribution factors than before, often causing up to a 27% reduction of liveload wheel lines applied to an interior girder.

LRFD MCF shear in its newest version may be slightly less conservative than some prior shear practice, especially for highly shear reinforced prestressed sections, and negative moment composite areas.

The following factors may be expected to cause prior LFD practices or designs to be less conservative than LRFD:

The HL 93 load is heavier than HS 20 or HS 25. This may effect Service I compressive stresses and Strength I, which will also be effected by the higher load factor for overlays.

C36: A balance between exterior girder design and interior has been achieved in some instances with an overhang of about 1' less than half the girder spacing.